

Lawrence Berkeley National Laboratory

Recent Work

Title

Lattice modeling of excavation damage in argillaceous clay formations: Influence of deformation and strength anisotropy

Permalink

<https://escholarship.org/uc/item/0c57v9hm>

Authors

Kim, K
Rutqvist, J
Birkholzer, J

Publication Date

2020-04-01

DOI

10.1016/j.tust.2019.103196

Peer reviewed

Lattice modeling of excavation damage in argillaceous clay formations: influence of deformation and strength anisotropy

Kunhwi Kim^{a,*}, Jonny Rutqvist^a, Jens Birkholzer^a

*^aEarth & Environmental Sciences Area, Lawrence Berkeley National Laboratory
1 Cyclotron Road, Berkeley, CA 94720, USA*

Abstract

This paper presents modeling of mechanical anisotropy in argillaceous rocks using an irregular lattice modeling approach, namely the rigid-body-spring network. To represent the mechanical anisotropy, new schemes are implemented in the modeling framework. The directionality of elastic deformation is resolved by modifying the element formulation with anisotropic elastic properties. The anisotropy of strength and failure characteristics is facilitated by adopting orientation-dependent failure criteria into the failure model. The verification of the improved modeling procedures is performed against theoretical model predictions for unconfined compression tests with various bedding orientations. Furthermore, excavation damage and fracturing processes in rock formations are simulated for different geomechanical configurations, such as rock anisotropy and tectonic heterogeneity. The simulated excavation damage characteristics are realistic and comparable with the actual field observation at a tunnel located in an argillaceous clay formation. The simulation results provide insights into the excavation damage zone

*Corresponding author.

Email address: KunhwiKim@lbl.gov (Kunhwi Kim)

phenomena with an explicit representation of fracturing processes.

Keywords: Mechanical anisotropy, Argillaceous clay formation, Excavation damage zone, Fracture modeling, Rigid-body-spring network.

1. Introduction

The investigation of damage development around underground excavations is a key issue in a variety of geoengineering fields, including mining, tunneling, and nuclear waste disposal (Bäckblom and Martin, 1999; Tsang et al., 2005; Hudson et al., 2009). Similar stability problems of underground openings, such as well boreholes, are crucial in the reservoir engineering and drilling industry related to hydrocarbon extraction and geothermal energy exploitation (Zoback, 2007). Especially, in the field of deep geological disposal of nuclear waste, the excavation damage zone (EDZ) may have an adverse impact on the mechanical and hydrological properties of rock mass, thus affecting the isolation performance, and furthermore, the long-term safety of the repository (Levasseur et al., 2010).

Argillaceous clay formations are considered as potential host or cap rocks for deep geological disposal of nuclear waste due to their superior isolation and confinement characteristics. These characteristics include low permeability and diffusivity, high retention capacity for radionuclides, self-sealing features, and long-term stability of the geological environment (Marschall et al., 2006; Blümling et al., 2007). However, special geomechanical conditions in rock formations, such as low strength and mechanical anisotropy and unfavorable high degree of anisotropy of in-situ stresses at depth, increase vulnerability to excavation damage and fracture development around a tunnel or a shaft, which often brings about serious engineering difficulties during the construction and operation of deep un-

22 derground repositories (Steiner, 1996; Martin et al., 1999; Bossart et al., 2004).
23 Therefore, rational understanding of the geomechanical settings and the corre-
24 sponding EDZ phenomena could help conducting more reliable performance and
25 safety assessment of the deep clay-based repositories.

26 The mechanical behavior of argillaceous clay formations, such as shale, is
27 greatly influenced by mechanical anisotropy, which is typically observed as a
28 variation of elastic response, strength characteristics, and failure mechanisms with
29 sample bedding orientation in laboratory experiments (McLamore and Gray, 1967;
30 Niandou et al., 1997; Naumann et al., 2007). At a larger scale, this mechanical
31 anisotropy directly affects the stability of underground structures and the observed
32 failure behavior. Field observations from the Mont Terri underground research
33 laboratory (URL) located in an indurated, over-consolidated clay shale, namely
34 Opalinus Clay, indicate that the geometry and extent of the EDZ around tunnels
35 are largely dependent on the relative orientation between bedding planes and the
36 excavation axis (Marschall et al., 2006; Blümling et al., 2007; Labiouse and Vi-
37 etor, 2014).

38 In this study, the mechanical anisotropy of argillaceous rocks is modeled within
39 the rigid-body-spring network (RBSN) framework. A rock mass is rendered as a
40 network of numerous spring sets connected with distinct one-dimensional ele-
41 ments (i.e., lattice elements), and the global mechanical behavior, including dis-
42 crete fracturing process, is represented by the collection of local deformation and
43 breakage of the spring sets. To represent the mechanical anisotropy within the
44 modeling procedures, formulation of the element stiffness matrix is modified with
45 anisotropic elastic configurations, and orientation-dependent failure criteria are
46 introduced. Details of the proposed modeling schemes are presented in Section 2.

47 The improved modeling procedures are applied to simulate unconfined compres-
48 sion tests in Section 3, where variations of Young’s modulus, uniaxial compres-
49 sive strength, and fracture pattern with the bedding orientation are evaluated and
50 verified against the theoretical model predictions. Furthermore, in Section 4, sim-
51 ulations of the EDZ evolutions are conducted for different geomechanical settings,
52 and comparisons of the simulated EDZ features are made as a validation process.

53 **2. Representation of mechanical anisotropy**

54 The rigid-body-spring network (RBSN) approach has been used to investigate
55 mechanics and fracture processes of geological systems, for which a rock mass is
56 modeled as an assemblage of simple, two-node elements in a lattice structure. The
57 irregular geometry of the lattice structure is defined by the dual Delaunay–Voronoi
58 tesellation of a set of nodal points randomly generated in the domain (Okabe et al.,
59 2000). This section presents methodologies for implementation of deformation
60 and strength anisotropy in the lattice model.

61 *2.1. Anisotropy of elastic deformability*

62 Figure 1 compares the formation of ordinary lattice elements and their modi-
63 fication for anisotropic elasticity. A 2D case is herein illustrated for plain descrip-
64 tion, but this scheme has been developed within the 3D modeling framework. A
65 lattice element is formed from two neighboring nodes, i and j , which are con-
66 nected via rigid-body constraints to a zero-size spring set located at the center of
67 the common Voronoi facet.

68 In the ordinary lattice model, the spring sets are oriented to their individual
69 local x – y coordinates defined by the Voronoi diagram (Fig. 1a). The spring coef-

70 ficients are defined as (Kawai, 1978)

$$\begin{aligned}
 k_x &= \alpha_1 E \frac{A_{ij}}{h_{ij}} \\
 k_y &= \alpha_2 k_x \\
 k_\phi &= E \frac{I_\phi}{h_{ij}}
 \end{aligned}
 \tag{1}$$

71 where A_{ij} is the area of the common Voronoi facet; h_{ij} is the distance between the
 72 element nodes; and I_ϕ is the moment of inertia of the facet area. Factors α_1 and α_2
 73 are adjusted to obtain global representation of Young's modulus E and Poisson's
 74 ratio ν . For the special case of $\alpha_1 = \alpha_2 = 1$, the lattice model provides an elasti-
 75 cally homogeneous and isotropic representation of E , although the corresponding
 76 value of $\nu = 0$ (Bolander and Saito, 1998; Yip et al., 2005). In the new scheme for
 77 anisotropic representation, by comparison, the spring sets are aligned to the direc-
 78 tion of bedding planes in N – P coordinates (Fig. 1b). The stiffness coefficients in
 79 Eq. 1 can be modified for anisotropic elastic properties about the orthogonal N –
 80 and P –axes:

$$\begin{aligned}
 k_N &= E_N \frac{A_{ij}}{h_{ij}} \\
 k_P &= E_P \frac{A_{ij}}{h_{ij}} \\
 k_\varphi &= E_N \frac{I_\phi}{h_{ij}}
 \end{aligned}
 \tag{2}$$

81 where E_N and E_P are Young's moduli normal and parallel to bedding, respectively,
 82 which can be directly adopted from laboratory measurements.

83 As seen in Fig. 1b, the kinematics of the nodes and elements can be repre-
 84 sented in three distinct coordinate systems: global X – Y coordinates based on do-
 85 main construction; local x – y coordinates for individual elements; and global N – P
 86 coordinates related to the bedding orientation. The anisotropic material matrix

87 $\mathbf{D} = \text{diag}[k_N, k_P, k_\varphi]$ for each lattice element is established in $N-P$ coordinates,
 88 then transformed to the spring set stiffness matrix in local $x-y$ coordinates using
 89 the coordinate transformation (McGuire and Gallagher, 1979):

$$\mathbf{k}_s = \mathbf{\Gamma}^T \mathbf{D} \mathbf{\Gamma} \quad (3)$$

90 where $\mathbf{\Gamma}$ is the 3×3 coordinate transformation matrix from $x-y$ coordinates to
 91 $N-P$ coordinates:

$$\mathbf{\Gamma} = \begin{bmatrix} N_x & N_y & 0 \\ P_x & P_y & 0 \\ 0 & 0 & 1 \end{bmatrix} \quad (4)$$

92 The first 2×2 entries in γ are the direction cosines between the bedding orientation
 93 and the local element axes.

94 The element stiffness matrix \mathbf{k}_e , relating the generalized local forces and el-
 95 ement nodal displacements, is derived from \mathbf{k}_s pre- and post-multiplied by the
 96 geometric matrix (Bolander and Saito, 1998; Berton and Bolander, 2006):

$$\mathbf{k}_e = \mathbf{B}^T \mathbf{k}_s \mathbf{B} \quad (5)$$

97 The conversion of \mathbf{k}_e to the global coordinates is obtained by another coordinate
 98 transformation:

$$\mathbf{K}_e = \mathbf{T}^T \mathbf{k}_e \mathbf{T} \quad (6)$$

99 where \mathbf{T} is the transformation matrix relating local $x-y$ and global $X-Y$ coor-
 100 dinate systems. The transformed element stiffness matrices, built for individual
 101 lattice elements, are assembled into the global stiffness matrix to solve the system
 102 equilibrium equations.

103 2.2. Anisotropy of strength

104 This study employs a Mohr–Coulomb model to determine brittle failure of lat-
105 tice elements. The stress state of each lattice element is represented as a Mohr cir-
106 cle in the stress space, which is assessed by a limiting surface for possible failure
107 conditions. Herein, a weak-plane failure model is used to provide anisotropic fail-
108 ure characteristics. This model assumes that the strength anisotropy is attributed
109 to conditional failure on a particular orientation of a plane, where the material
110 strength is significantly weaker. This model concept has a physical basis because
111 the bedding planes in sedimentary rocks are generally recognized as planes of
112 weakness (Pariseau, 2006; Fjær et al., 2008).

113 Figure 2 illustrates two distinct situations of orientation-dependent failure in
114 the Mohr–Coulomb model. The material has two failure criteria—one intrinsic,
115 isotropic criterion and one for the weak planes—and correspondingly two failure
116 surfaces (Jaeger, 1960). The intrinsic failure criterion is given by the cohesive
117 strength c , internal friction angle ϕ , and tension cut-off f_t . Analogously, the weak-
118 plane failure criterion is defined by lower values of the strength parameters c_w , ϕ_w ,
119 and $f_{t,w}$. Consider the stress state in the rock specimen is such that the Mohr circle
120 intersects the weak-plane failure surface at two points with angles $2\psi_1$ and $2\psi_2$
121 (Fig. 2a), and the material with a bedding orientation $\psi_1 < \beta < \psi_2$ will fail along
122 the bedding planes at a lower stress level. However, if the bedding orientation β is
123 projected below the weak-plane failure surface, as shown in Fig. 2b, the material
124 will fail across the bedding planes with the failure angle $\psi = 45^\circ + \phi/2$ at a higher
125 stress level.

126 For the confined stress configuration ($\sigma_1 > \sigma_2 = \sigma_3 > 0$), the two failure
127 criteria limit the stress states, which can be theretically defined as follows (Jaeger

et al., 2007):

$$\sigma_1 - \sigma_3 = 2 \frac{c \cos \phi + \sigma_3 \sin \phi}{1 - \sin \phi} \quad (7)$$

for the intrinsic failure criterion; and

$$\sigma_1 - \sigma_3 = 2 \frac{c_w \cos \phi_w + \sigma_3 \sin \phi_w}{\sin 2\beta \cos \phi_w - (1 + \cos 2\beta) \sin \phi_w} \quad (8)$$

for the weak-plane failure criterion. The criterion that predicts the lowest strength for a given orientation β is always the relevant criterion in the failure model. In that sense, the anisotropic strength could vary with the bedding orientation of the sample.

3. Unconfined compression tests

The proposed modeling schemes are verified in the simulations of unconfined compression tests for transversely isotropic rock specimens. Consider a cylindrical core sample subjected to a uniaxial compression load, in which the bedding planes form an angle θ from the axial loading direction (see Fig. 3a). We have conducted simulations for eight cases of angles to bedding: $\theta = -15^\circ$; 0° (parallel to loading axis); 15° ; 30° ; 45° ; 60° ; 75° ; and 90° (normal to loading axis). As shown in Fig. 3b, a core sample with a diameter of 50 mm and a height of 100 mm is discretized with 15,047 nodes and 76,730 elements. Extra layers of nodes and elements are padded at the top and bottom of the cylinder to provide uniform axial strain along the section, where a displacement controlled boundary condition is applied to deform the sample up to 0.3 mm in compression. The anisotropic properties of the core material are adopted from the experimental results for the Opalinus Clay samples (Bossart, 2011), which are listed in Table 1.

148 The resulting stress-strain curves for the eight cases of bedding orientations
 149 are plotted in Fig. 4. The stress response is calculated by averaging the sum of
 150 reaction forces monitored at the padding layers over the cross sectional area of the
 151 sample. The height of the core sample is taken as the gauge length to derive global
 152 axial strain from the boundary displacements. The stress linearly increases with
 153 the strain to the peak stress and sharply decreases over the peak, involving brittle
 154 failure. The linear slope and the peak of the response curve vary with the bedding
 155 orientation, which reflects the mechanical anisotropy of the rock material. Note
 156 that the response curves for $\theta = -15^\circ$ and 15° coincide closely with each other
 157 because of the symmetric bedding orientations with the same angle from the
 158 loading axis.

159 The linear slope of each stress-strain curve is taken as the global Young's mod-
 160 ulus of the sample and compared with a theory in Fig. 5. The Young's modulus
 161 of a transversely isotropic material with an angle θ from the loading axis to the
 162 bedding is theoretically obtained by Pariseau (2006):

$$\frac{1}{E} = \frac{\cos^4 \theta}{E_P} + \left(\frac{1}{G_a} - \frac{2\nu_a}{E_P} \right) \sin^2 \theta \cos^2 \theta + \frac{\sin^4 \theta}{E_N} \quad (9)$$

163 where G_a and ν_a are the relevant shear modulus and Poisson's ratio for the anisotropic
 164 case, respectively. With an approximation of $G_a = 1 / \left(\frac{1}{E_P} + \frac{1}{E_N} \right)$ for zero Poisson's
 165 ratio, Eq. 9 can be rewritten as

$$\frac{1}{E} = \frac{\cos^2 \theta}{E_P} + \frac{\sin^2 \theta}{E_N} \quad (10)$$

166 The variation of Young's modulus from the simulations shows a good agreement
 167 with the theoretical model prediction.

168 The peak stress captured from the stress-strain response can be regarded as
 169 the uniaxial compressive strength. Figure 6 shows the simulated uniaxial com-

170 pressive strengths for the cases of different angles to bedding. The orientation-
 171 dependent failure criteria presented in Section 2.2 limit the critical stress state of
 172 the anisotropic material. By imposing the lateral unconfined condition ($\sigma_3 = 0$)
 173 and substituting the relevant strength parameters into Eqs. 7 and 8, the theoretical
 174 uniaxial compressive strength at an angle to bedding $\theta = 90^\circ - \beta$ can be determined
 175 as the lowest strength between

$$UCS = \frac{2c_P \cos \phi_P}{1 - \sin \phi_P} \quad (11)$$

176 and

$$UCS = \frac{2c_N \cos \phi_N}{\sin 2\theta \cos \phi_N - (1 - \cos 2\theta) \sin \phi_N} \quad (12)$$

177 This theoretical model prediction is also plotted as a function of the angle to bed-
 178 ding θ in Fig. 6, which is perfectly in line with the simulated uniaxial compressive
 179 strengths. However, the material exhibits anisotropic strengths only when the bed-
 180 ding orientations fall in a certain range for the weak-plane failure and otherwise a
 181 constant strength involved with the intrinsic isotropic failure, which is attributed
 182 to the discontinuous set of a single plane of weakness in the failure model. More
 183 sophisticated models such that weak planes and the corresponding strength
 184 parameters vary continuously with the bedding orientation could provide more
 185 realistic representation of rock anisotropy (McLamore and Gray, 1967; Hoek and
 186 Brown, 1980; Pietruszczak and Mroz, 2001).

187 Figure 7 presents fracture patterns within the samples at the final loading stage
 188 (0.003 of axial strain). Overall, cracks tend to propagate in the direction of bed-
 189 ding planes, especially for the cases of low inclination angles of bedding planes
 190 from the loading axis. In these cases, weak-plane failure along the bedding planes
 191 is selected as the relevant mechanism to proceed the material failure. A com-
 192 parison of Figs. 7a and c indicates that symmetric bedding orientations about the

loading axis naturally lead to symmetric patterns of fracture development. In the exceptional case with 45° to bedding (Fig. 7e), the fractures align perpendicular to the bedding planes, which form a thick shear band across the sample.

In the simulations of unconfined compression tests, the elastic deformability and failure characteristics of the anisotropic rock material are rationally demonstrated. Various bedding orientations of the core samples are represented without the need of orientation-dependent mesh geometry, but rather with inherent anisotropy of lattice elements. Also, the mechanical parameters are uniformly assigned to the elements irrespective of their local orientations, which contrasts to other modeling approaches using heterogeneous parameters dependent on the direction of element (Lisjak et al., 2014, 2015). In the next section, the modeling procedures are further validated through simulations of excavation damage and fracturing processes in rock formations.

4. Excavation damage and fracture development in argillaceous clay formations

4.1. HG-A microtunnel at the Mont Terri site

The evolution of excavation damage zone (EDZ) near a tunnel is simulated and validated against the field observation at the HG-A test site. A circular micro-tunnel with the length of 13 m and the diameter of 1.04 m is located at the Mont Terri URL near Saint-Ursanne, Switzerland. The host rock, Opalinus Clay, is relatively homogeneous in meter-scale, but pronounced anisotropy with bedding planes is discovered at millimeter-scale (Yong et al., 2010). The rock formation surrounding the HG-A tunnel is highly fractured with a sub-meter spatial frequency although the fracture permeability is not significant, which indicates that

217 fractures are mostly closed under natural stress conditions (Marschall et al., 2006,
218 2008).

219 One main purpose of the HG-A test is to provide data on the geomechanical
220 and hydrogeological effects due to the presence of the EDZ around the tunnel.
221 Although the long-term physical features observed in the test are related to cou-
222 pled hydro-mechanical processes, herein the problem is simplified by assuming
223 a constant uniform pore pressure such that the excavation damage and fractur-
224 ing processes can be reckoned as short-term mechanical-only responses. This
225 assumption is valid up until an early tunneling stage, where mechanical deforma-
226 tions occur much more quickly than water flow and hydrological processes in the
227 rock formation, and the mechanical equilibrium is held within a rapid (undrained)
228 excavation (Liu et al., 2013). In the actual excavation of the HG-A tunnel, the
229 drilling progress was rather quick and smooth (Marschall et al., 2006), so pure
230 mechanical simulations are applicable to this case, where relevant EDZ phenom-
231 ena could be captured.

232 Figure 8 shows excavation damage around the microtunnel. Anisotropic prop-
233 erties of the rock material and heterogeneity of tectonic fault formations, as well
234 as anisotropic in-situ stress conditions, result in non-uniform damage around the
235 tunnel. Major buckling and spalling phenomena between 10 and 11 o'clock
236 and less distinct buckling at about 5 o'clock are observed along the tunnel wall,
237 which indicates that the tunnel is running parallel to the strike of inclined bedding
238 planes (Marschall et al., 2006). On the other hand, wedge-shaped damage struc-
239 tures with extension joints mapped on the side surfaces (3 to 4 o'clock and 9 to 10
240 o'clock) are identified as stress-induced breakouts.

241 In this study, we investigate the effects of rock anisotropy and tectonic hetero-

242 geneity on the EDZ phenomena. Figure 9 presents three simulations cases with
 243 different geomechanical settings: (I) in-plane isotropy and intact formation; (II) in-
 244 plane anisotropy and intact formation; and (III) in-plane anisotropy and fault
 245 formation. Anisotropic in-situ stresses of $\sigma_v = 6.5$ MPa in the vertical direction
 246 and $\sigma_h = 4.5$ MPa in the horizontal direction are adopted as a confinement condi-
 247 tion (Martin and Lanyon, 2003), and a uniform pore pressure of 1.5 MPa is applied
 248 in the modeling domain. For Cases II and III, the bedding planes are oriented at
 249 45° from the horizontal axis (Figs. 9b and c), and additionally for Case III three
 250 discrete fault planes are placed around the tunnel (Fig. 9c).

251 Figure 10 depicts the Voronoi discretizations of a 10 m square domain for the
 252 excavation damage simulations. Predicted EDZ area around the tunnel is finely
 253 meshed, and the nodal density is graded towards the domain boundaries for com-
 254 putational efficiency. The circular tunnel with 1.04 m diameter is initially filled
 255 with the Voronoi cells and lattice elements. The excavation process is realized by
 256 gradually reducing the spring stiffnesses, internal element forces, and pore pres-
 257 sure of the elements to void the tunnel domain, for which an exponential decay is
 258 assumed to set the reduction of the values to 10^{-6} of the original values at the end
 259 of 100 loading steps. Herein, gravity loads are ignored throughout the simula-tions
 260 because the gravity forces may have a minor influence compared to the effect of
 261 in-situ stresses in case of relatively deep, small excavations (Carranza-Torres and
 262 Fairhurst, 1997). As shown Fig. 10b, fault planes are explicitly represented in the
 263 mesh, and low strength parameters are assigned to the corresponding el-ements:
 264 tensile strength $f_{t,f} = 0.5$ MPa; cohesion $c_f = 1.0$ MPa; and friction angle $\phi_f = 23^\circ$.
 265 Rock properties are set as listed in Table. 1. Note that while clay shale may exhibit
 266 plastic deformations or residual stress responses under confined

267 stress conditions (Parisio et al., 2015), this study simply assumes the rock material
268 around the tunnel opening undergoes brittle failure with unconfined conditions.

269 4.2. *Discussion on simulation results*

270 From the simulations of the EDZ evolutions, the resulting damage patterns
271 and the contours of the magnitudes of the major principal stress and the minor
272 principal stress are given in Figures 11 to 13. For Case I with isotropic rock
273 properties, excavation-induced cracks stretch out in the direction perpendicular
274 to the major confining stress to result in the formation extensive breakouts or v-
275 shaped notches (see Fig. 11a), which is a failure pattern frequently found under
276 anisotropic in-situ stress conditions (Martin et al., 1999; Read, 2004; Perras and
277 Diederichs, 2016). As seen in Figs. 11b and c, the in-situ stress field is altered
278 by the excavation. The failed zones exhibit low magnitudes of stresses due to
279 softening and weakening effects of the fractures, however, the redistribution of
280 stresses is highly concentrated close to the notch tips. In the outer region, the
281 disturbed stress field displays quite symmetric contours about the vertical and
282 horizontal axes crossing at the center of the tunnel.

283 The simulation results for Case II, presented in Fig. 12, show failure charac-
284 teristics influenced by the anisotropy of deformability and strength. As shown in
285 Fig. 12a, more pronounced cracks and breakouts at the tunnel wall are concen-
286 trated and oriented along the direction normal to the bedding planes. Compared
287 to the stress contours in the previous case, the contour patterns are asymmetric
288 and inclined towards the bedding orientation (see Figs. 12b and c), which may be
289 attributed to the anisotropic rock properties and the inclined failed zones.

290 For Case III with anisotropic rock properties and fault planes, the EDZ evolu-
291 tion involves more complex failure processes. Distinct shear failures occur along

the fault planes in the early stage of excavation, and then rock cracking grows from the fault planes. The failed zones illustrated in Fig. 13a conform to the estimation of damaged zones from the field observation (see Fig. 8). One notable feature in the fractured damage pattern is that the crack growth in the rock is somewhat controlled by the fault planes, so that the cracks do not propagate across the faults. Figures 13b and c show the stress field perturbed by the fault planes, where discontinuous stress contours are spotted in the fractured zones and extended along the fault planes.

5. Conclusions

The evolution of excavation damage zone (EDZ) and the failure features around the tunnel excavation are found to be strongly related to the lithological properties of the rock formations, i.e., rock anisotropy due to the bedding planes. In this study, the rigid-body-spring network (RBSN) approach is used to simulate the mechanically anisotropic behavior of argillaceous rocks. Since the original RBSN models generally provide the descriptions of isotropic systems, the modeling procedures have been modified and improved with new modeling schemes to represent the mechanical anisotropy. The spring coefficients of individual lattice elements are systematically formulated based on the bedding orientation, and in addition, a simple weak-plane failure model is adopted into the Mohr–Coulomb criteria to determine the local failure along the bedding planes. The improved modeling procedures can readily represent the anisotropic elasticity and effectively capture the orientation-dependent failure features and strength anisotropy.

Using the new schemes in the RBSN model, unconfined compression tests are simulated for transversely isotropic rock specimens, and the simulation results

are verified against analytical solutions. Various bedding orientations of the samples can be reflected on the identical mesh without modifying or re-constructing the mesh geometry. From the stress-strain responses for the simulated cases, the global Young's moduli and the uniaxial compressive strengths are evaluated, which fall in line with the analytically determined values of anisotropic properties. Moreover, anisotropic failure characteristics are observed from the orientation of fracture development relative to the direction of bedding planes.

Next, the modeling procedures are applied to simulate the tunnel excavation in rock formations, where three cases of geomechanical settings are considered to make a comparative study on the EDZ phenomena. Several observations can be highlighted:

- The EDZ evolution in the isotropic rock formation is largely influenced by the in-situ stress field. Major failure and extensive breakouts are generated to build v-shaped notches from the tunnel wall in the direction perpendicular to the major principal compressive stress.
- In the presence of rock anisotropy, excavation-induced fractures are inclined to develop on the tunnel walls tangential to the bedding planes. Due to the anisotropic elastic properties, stress contours around the tunnel are asymmetric about the principal axis of in-situ stress and inclined to the bedding orientation.
- Tectonic faults can contribute to the heterogeneous excavation damage, where distinct shear failures occur along the fault planes and the stress contours are divided into discontinuous regions. The resulting fracture pattern in the damaged zones is also affected by the fault planes.

340 In summary, the simulations presented in this paper confirm the validity of the
341 RBSN approach to model the EDZ evolutions in argillaceous rocks. The EDZ
342 phenomena reproduced in the simulations are realistic and comparable with the
343 actual EDZ observation at the HG-A microtunnel. The simulation results provide
344 insights into the characterizations of excavation damage with an explicit represen-
345 tation of fracturing processes.

346 **Acknowledgment**

347 This research is supported by the Spent Fuel and Waste Disposition Campaign,
348 Office of Nuclear Energy of the U.S. Department of Energy, under Contract Num-
349 ber DE-AC02- 05CH11231 with Lawrence Berkeley National Laboratory.

350 **References**

- 351 Bäckblom, G., Martin, C. D., 1999. Recent experiments in hard rocks to study
352 the excavation response: Implications for the performance of a nuclear waste
353 geological repository. *Tunnelling and Underground Space Technology* 14 (3),
354 377–394.
- 355 Berton, S., Bolander, J. E., 2006. Crack band model of fracture in irregular lattices.
356 *Computer Methods in Applied Mechanics and Engineering* 195 (52), 7172–
357 7181.
- 358 Blümling, P., Bernier, F., Lebon, P., Martin, C. D., 2007. The excavation dam-
359 aged zone in clay formations time-dependent behaviour and influence on per-
360 formance assessment. *Physics and Chemistry of the Earth, Parts A/B/C* 32 (8),
361 588–599.

- 362 Bolander, J. E., Saito, S., 1998. Fracture analyses using spring networks with
363 random geometry. *Engineering Fracture Mechanics* 61 (5-6), 569–591.
- 364 Bossart, P., 2011. Characteristics of the Opalinus Clay at Mont Terri. Mont Terri
365 Project, Wabern, Switzerland.
- 366 Bossart, P., Trick, T., Meier, P. M., Mayor, J.-C., 2004. Structural and hydrogeo-
367 logical characterisation of the excavation-disturbed zone in the Opalinus Clay
368 (Mont Terri Project, Switzerland). *Applied Clay Science* 26, 429–448.
- 369 Carranza-Torres, C., Fairhurst, C., 1997. On the stability of tunnels under gravity
370 loading, with post-peak softening of the ground. *International Journal of Rock*
371 *Mechanics and Mining Sciences* 34 (3–4), 75.e1–75.e18.
- 372 Fjær, E., Holt, R. M., Horsrud, P., Raaen, A. M., Risnes, R. (Eds.), 2008.
373 *Petroleum Related Rock Mechanics*, 2nd Edition. Vol. 53 of *Developments in*
374 *Petroleum Science*. Elsevier.
- 375 Hoek, E., Brown, E. T., 1980. Empirical strength criterion for rock masses. *Journal*
376 *of Geotechnical and Geoenvironmental Engineering* 106, 1013–1035.
- 377 Hudson, J. A., Bäckström, A., Rutqvist, J., Jing, L., Backers, T., Chijimatsu,
378 M., Christiansson, R., Feng, X.-T., Kobayashi, A., Koyama, T., Lee, H.-S.,
379 Neretnieks, I., Pan, P.-Z., Rinne, M., Shen, B.-T., 2009. Characterising and
380 modelling the excavation damaged zone in crystalline rock in the context of
381 radioactive waste disposal. *Environmental Geology* 57 (6), 1275–1297.
- 382 Jaeger, J. C., 1960. Shear failure of anisotropic rocks. *Geological Magazine* 97 (1),
383 65–72.

- 384 Jaeger, J. C., Cook, N. G. W., Zimmerman, R., 2007. Fundamentals of Rock Me-
385 chanics, 4th Edition. Wiley-Blackwell.
- 386 Kawai, T., 1978. New discrete models and their application to seismic response
387 analysis of structures. Nuclear Engineering and Design 48, 207–229.
- 388 Labiouse, V., Vietor, T., 2014. Laboratory and in situ simulation tests of the exca-
389 vation damaged zone around galleries in Opalinus Clay. Rock Mechanics and
390 Rock Engineering 47 (1), 57–70.
- 391 Levasseur, S., Charlier, R., Frieg, B., Collin, F., 2010. Hydro-mechanical mod-
392 elling of the excavation damaged zone around an underground excavation at
393 Mont Terri Rock Laboratory. International Journal of Rock Mechanics and
394 Mining Sciences 47 (3), 414–425.
- 395 Lisjak, A., Garitte, B., Grasselli, G., Müller, H., Vietor, T., 2015. The excava-
396 tion of a circular tunnel in a bedded argillaceous rock (Opalinus Clay): Short-
397 term rock mass response and FDEM numerical analysis. Tunnelling and Un-
398 derground Space Technology 45, 227–248.
- 399 Lisjak, A., Grasselli, G., Vietor, T., 2014. Continuum–discontinuum analysis of
400 failure mechanisms around unsupported circular excavations in anisotropic clay
401 shales. International Journal of Rock Mechanics and Mining Sciences 65, 96–
402 115.
- 403 Liu, H. H., Houseworth, J., Rutqvist, J., Zheng, L., Asahina, D., Li, L., Vilarrasa,
404 V., Chen, F., Nakagawa, S., Finsterle, S., Doughty, C., Kneafsey, T., Birkholzer,
405 J., 2013. Report on thmc modeling of the near field evolution of a generic clay

406 repository: Model validation and demonstration. Tech. Rep. FCRD-UFD-2013-
 407 000244, Lawrence Berkeley National Laboratory, Berkeley, CA.

408 Marschall, P., Distinguin, M., Shao, H., Bossart, P., Enachescu, C., Trick, T.,
 409 2006. Creation and evolution of damage zones around a microtunnel in a clay-
 410 stone formation of the Swiss Jura Mountains. In: SPE International Symposium
 411 and Exhibition on Formation Damage Control. No. SPE-98537-PP. Society of
 412 Petroleum Engineers.

413 Marschall, P., Trick, T., Lanyon, G. W., Delay, J., Shao, H., 2008. Hydro-
 414 mechanical evolution of damaged zones around a microtunnel in a claystone
 415 formation of the Swiss Jura Mountains. In: The 42nd US Rock Mechanics
 416 Symposium (USRMS). No. ARMA-08-193. American Rock Mechanics Asso-
 417 ciation.

418 Martin, C. D., Kaiser, P. K., McCreath, D. R., 1999. Hoek-Brown parameters for
 419 predicting the depth of brittle failure around tunnels. Canadian Geotechnical
 420 Journal 36 (1), 136–151.

421 Martin, C. D., Lanyon, G. W., 2003. Measurement of in-situ stress in weak rocks
 422 at Mont Terri Rock Laboratory, Switzerland. International Journal of Rock Me-
 423 chanics and Mining Sciences 40 (7), 1077–1088.

424 McGuire, W., Gallagher, R. H., 1979. Matrix Structural Analysis. John Wiley &
 425 Sons, New York.

426 McLamore, R., Gray, K. E., 1967. The mechanical behavior of anisotropic sedi-
 427 mentary rocks. Journal of Engineering for Industry 89 (1), 62–73.

- 428 Naumann, M., Hunsche, U., Schulze, O., 2007. Experimental investigations on
429 anisotropy in dilatancy, failure and creep of Opalinus Clay. *Physics and Chem-
430 istry of the Earth, Parts A/B/C* 32 (8), 889–895.
- 431 Niandou, H., Shao, J. F., Henry, J. P., Fourmaintraux, D., 1997. Laboratory inves-
432 tigation of the mechanical behaviour of Tournemire shale. *International Journal
433 of Rock Mechanics and Mining Sciences* 34 (1), 3–16.
- 434 Okabe, A., Boots, B., Sugihara, K., Chiu, S. N., 2000. *Spatial Tessellations: Con-
435 cepts and Applications of Voronoi Diagrams*. John Wiley & Sons.
- 436 Pariseau, W. G., 2006. *Design Analysis in Rock Mechanics*. CRC Press, London,
437 UK.
- 438 Parisio, F., Samat, S., Laloui, L., 2015. Constitutive analysis of shale: a coupled
439 damage plasticity approach. *International Journal of Solids and Structures* 75-
440 76, 88–98.
- 441 Perras, M. A., Diederichs, M. S., 2016. Predicting excavation damage zone depths
442 in brittle rocks. *Journal of Rock Mechanics and Geotechnical Engineering* 8 (1),
443 60–74.
- 444 Pietruszczak, S., Mroz, Z., 2001. On failure criteria for anisotropic cohesive-
445 frictional materials. *International Journal for Numerical and Analytical Meth-
446 ods in Geomechanics* 25 (5), 509–524.
- 447 Read, R. S., 2004. 20 years of excavation response studies at AECL’s Under-
448 ground Research Laboratory. *International Journal of Rock Mechanics and
449 Mining Sciences* 41 (8), 1251–1275.

- 450 Steiner, W., 1996. Tunnelling in squeezing rocks: Case histories. *Rock Mechanics*
451 *and Rock Engineering* 29 (4), 211–246.
- 452 Tsang, C.-F., Bernier, F., Davies, C., 2005. Geohydromechanical processes in
453 the Excavation Damaged Zone in crystalline rock, rock salt, and indurated and
454 plastic clays—in the context of radioactive waste disposal. *International Journal*
455 *of Rock Mechanics and Mining Sciences* 42 (1), 109–125.
- 456 Yip, M., Mohle, J., Bolander, J. E., 2005. Automated modeling of three-
457 dimensional structural components using irregular lattices. *Computer-Aided*
458 *Civil and Infrastructure Engineering* 20 (6), 393–407.
- 459 Yong, S., Kaiser, P. K., Loew, S., 2010. Influence of tectonic shears on tunnel-
460 induced fracturing. *International Journal of Rock Mechanics and Mining Sci-*
461 *ences* 47 (6), 894–907.
- 462 Zoback, M. D., 2007. *Reservoir Geomechanics*. Cambridge University Press.

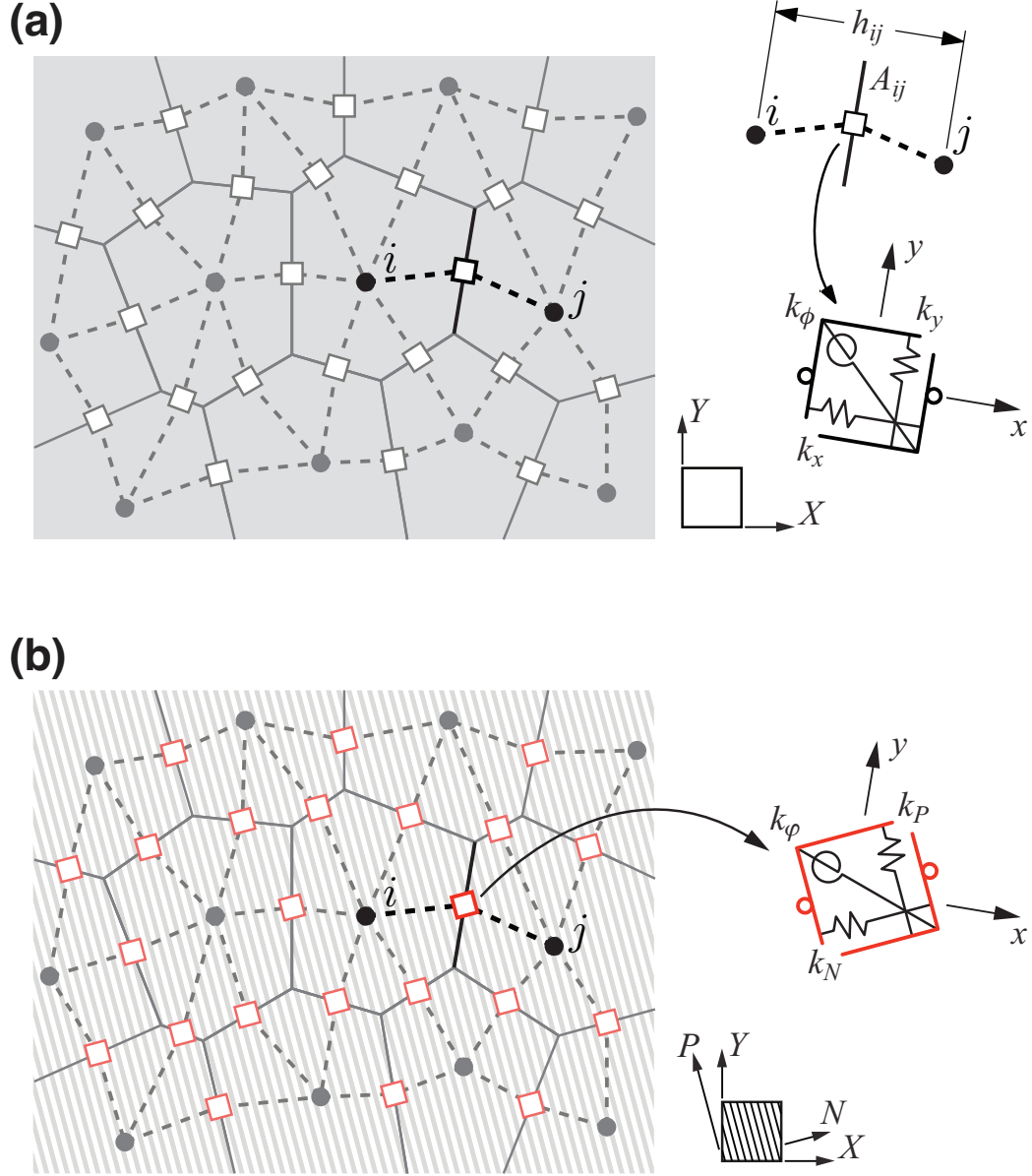


Figure 1: Modification of rigid-body-spring elements in an identical lattice structure: (a) original spring sets based on the direction of Voronoi facets (local x - y coordinate systems); and (b) spring sets aligned to the global bedding orientation (N - P coordinate system).

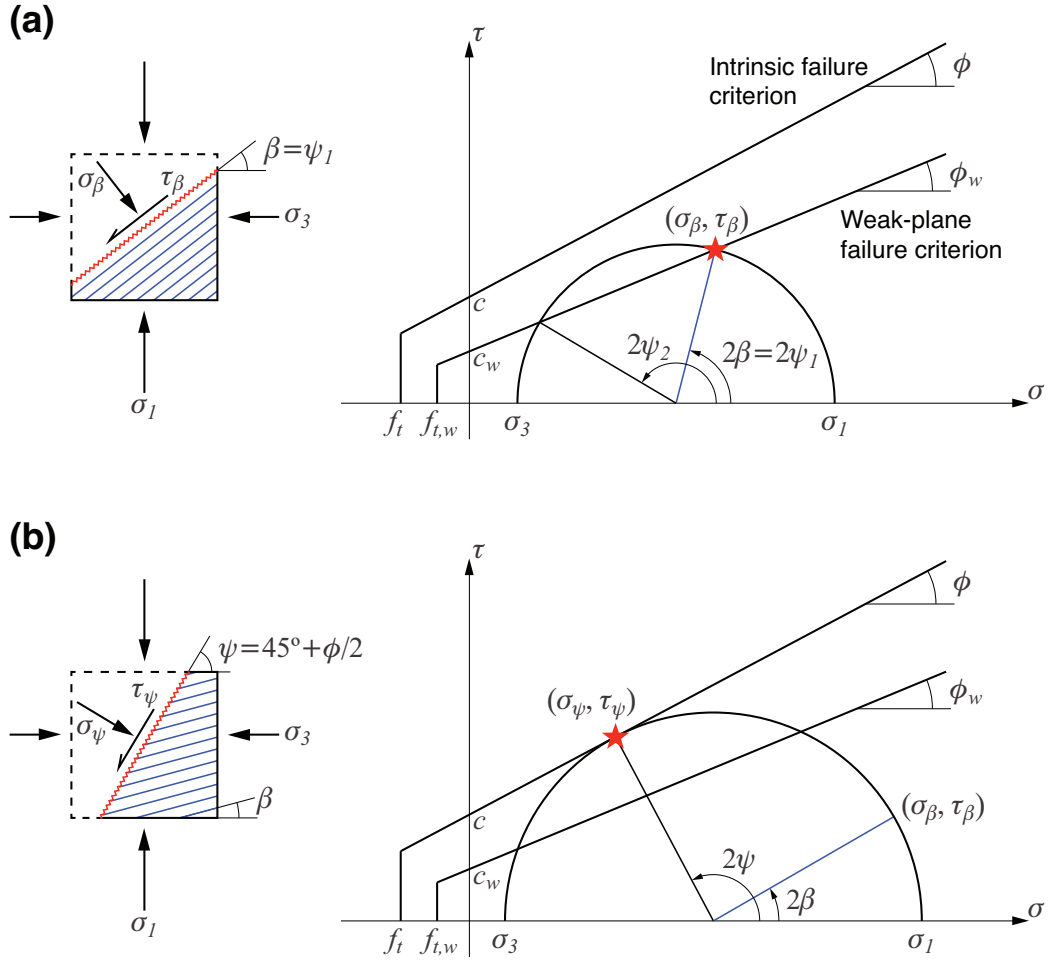


Figure 2: Orientation-dependent failure criteria in Mohr–Coulomb model: (a) weak-plane failure along the bedding; and (b) intrinsic failure across the bedding.

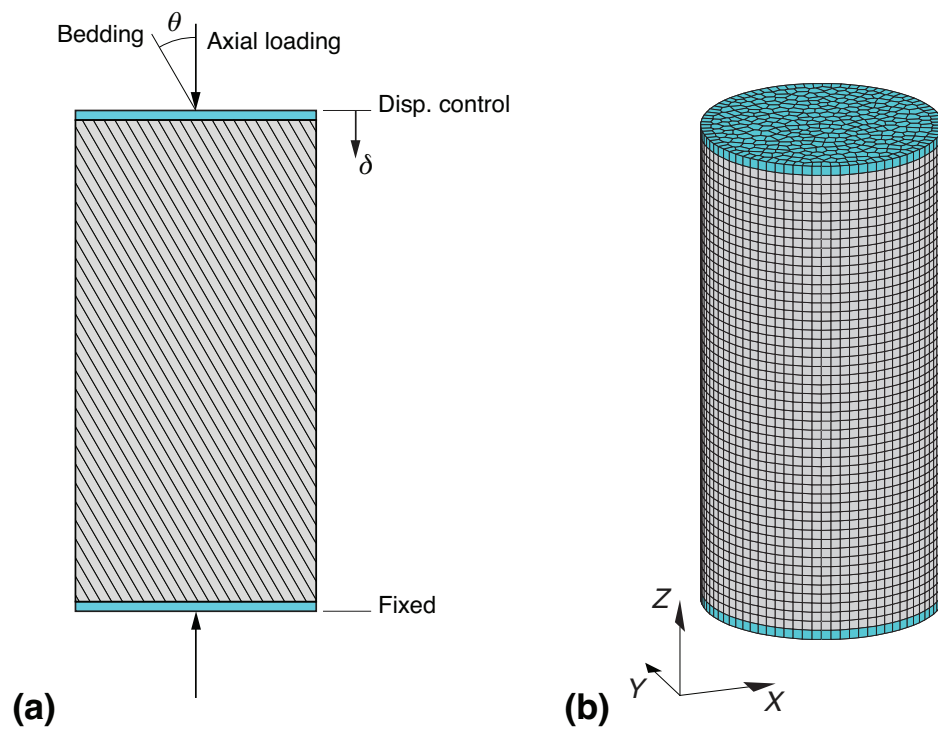


Figure 3: Uniaxial compression tests: (a) schematic drawing of the test program; and (b) 3D Voronoi discretization of a specimen model.

Table 1: Anisotropic mechanical properties of Opalinus Clay (Bossart, 2011)

Rock properties	Parallel to bedding	Normal to bedding
Young's modulus	$E_P = 15.5 \text{ GPa}$	$E_N = 9.5 \text{ GPa}$
Uniaxial tensile strength	$f_{t,P} = 2.0 \text{ MPa}$	$f_{t,N} = 1.0 \text{ MPa}$
Cohesion [†]	$c_P = 5.5 \text{ MPa}$	$c_N = 2.2 \text{ MPa}$
Internal friction angle	$\phi_P = 25^\circ$	$\phi_N = 25^\circ$

[†] Bossart (2011) suggested three different values for cohesion or shear strength: maximum value of 5.5 MPa (parallel to bedding); minimum value of 2.2 MPa (normal to bedding); and the third value of 1 MPa (shear strength of bedding planes). In this study, 2.2 MPa is taken as a cohesive strength normal to bedding, as stated.

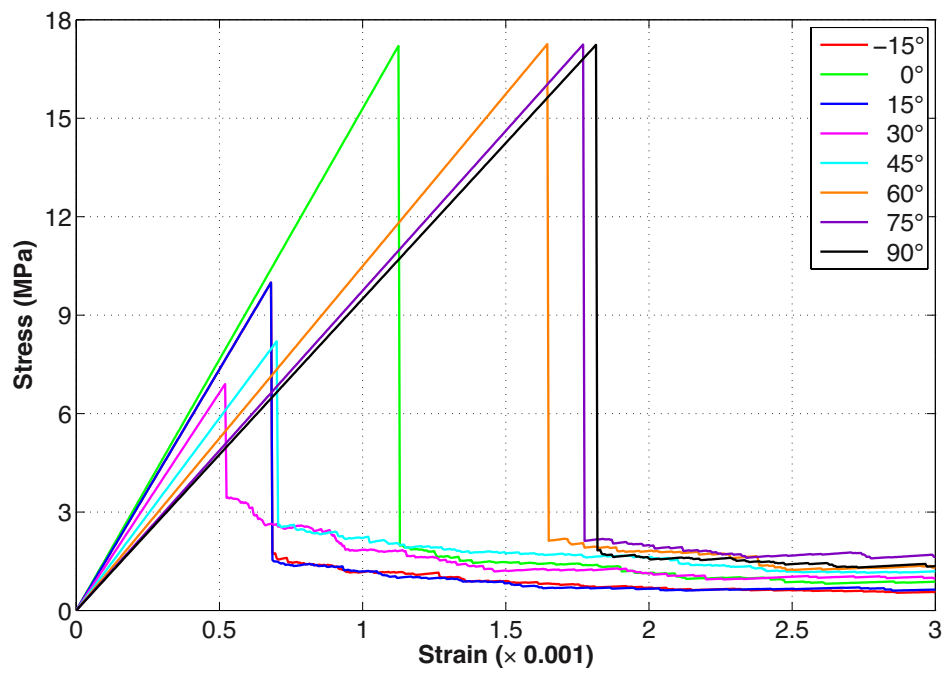


Figure 4: Stress-strain responses for various angles to bedding.

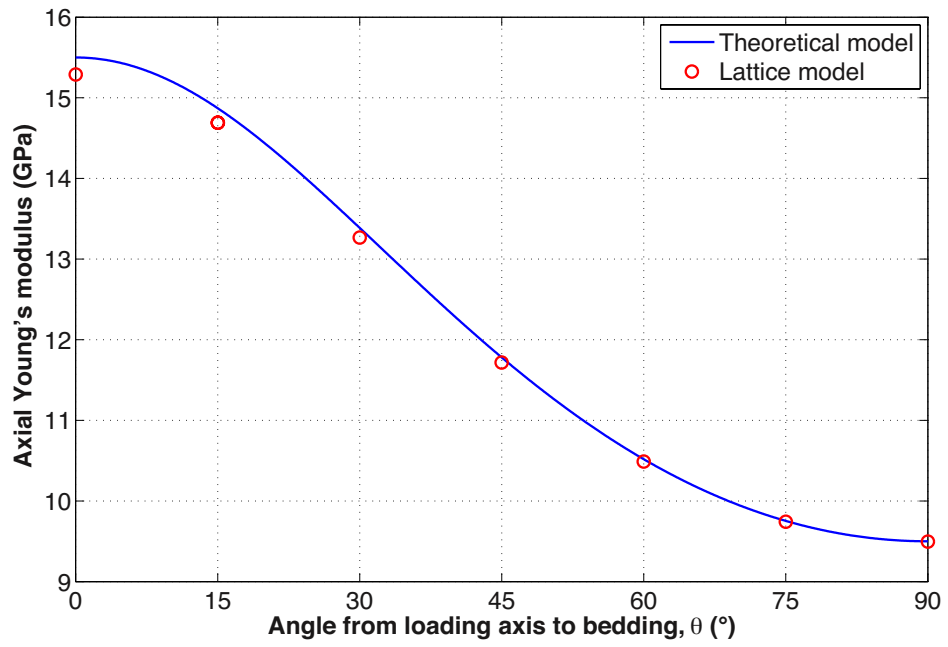


Figure 5: Comparison of simulated Young's modulus with the theoretical model prediction (Eq. 10) for different bedding orientations.

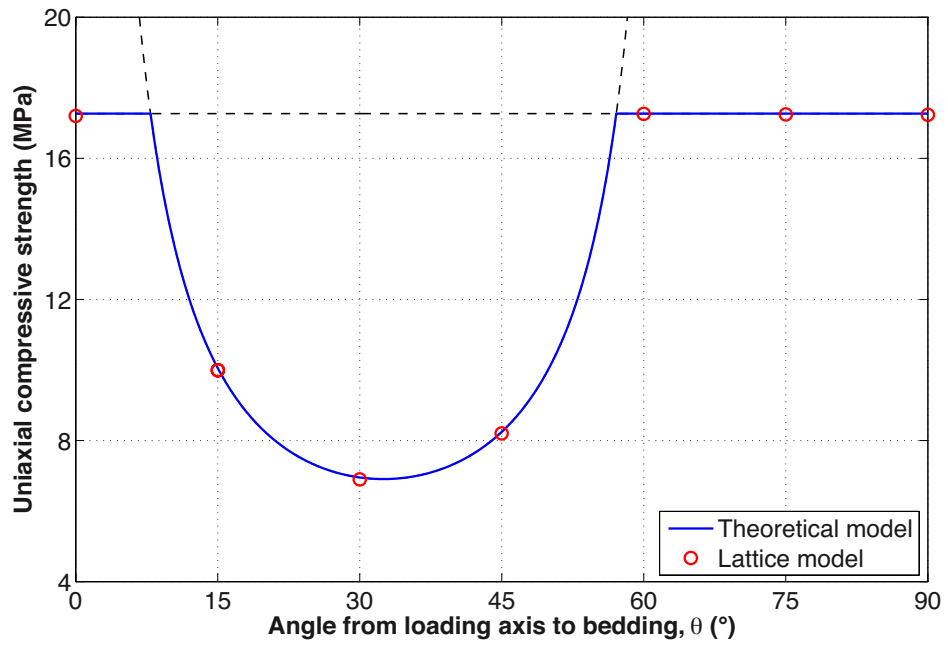


Figure 6: Comparison of simulated uniaxial compressive strength with the theoretical model prediction (determined by Eqs. 11 and 12) for different bedding orientations.

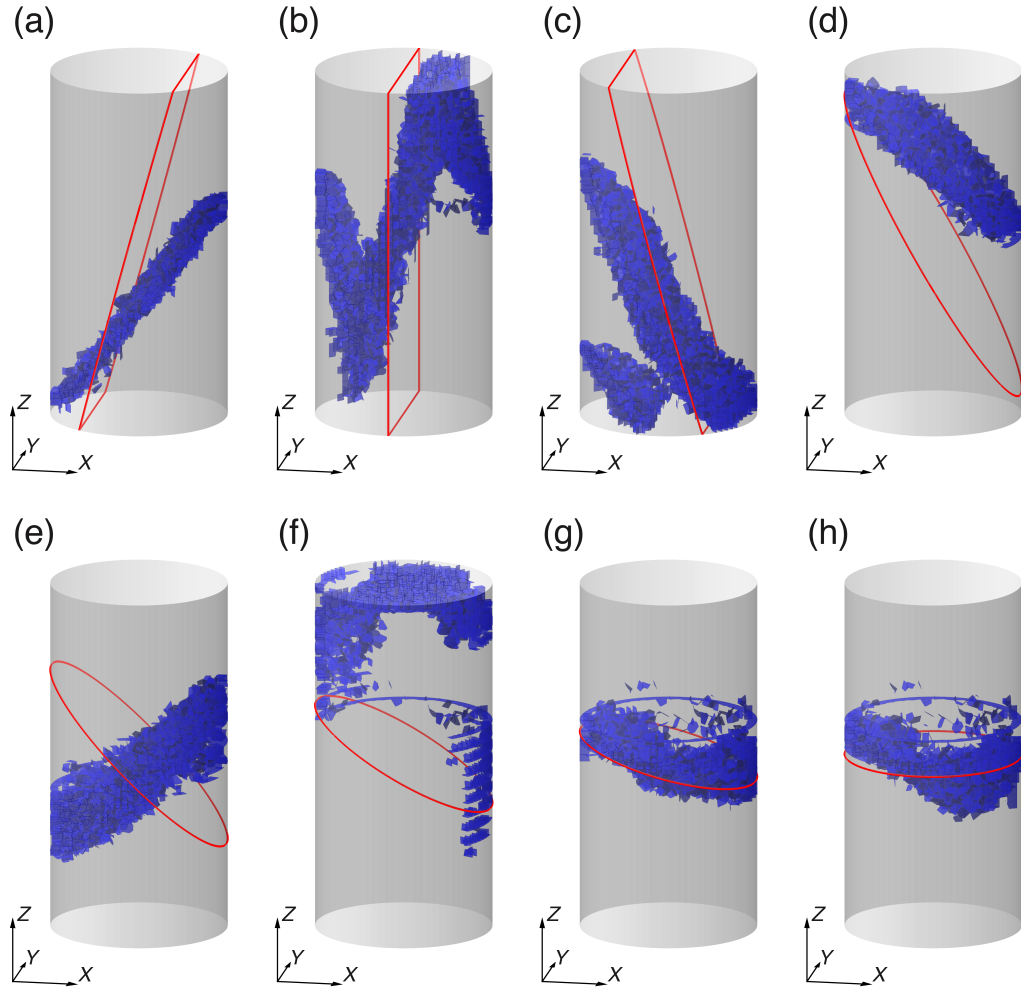


Figure 7: Comparison of fracture patterns for angles to bedding, $\theta =$ (a) -15° ; (b) 0° ; (c) 15° ; (d) 30° ; (e) 45° ; (f) 60° ; (g) 75° ; and (h) 90° . Note that the red contour of cylindric section for each case refers to the representative bedding plane.

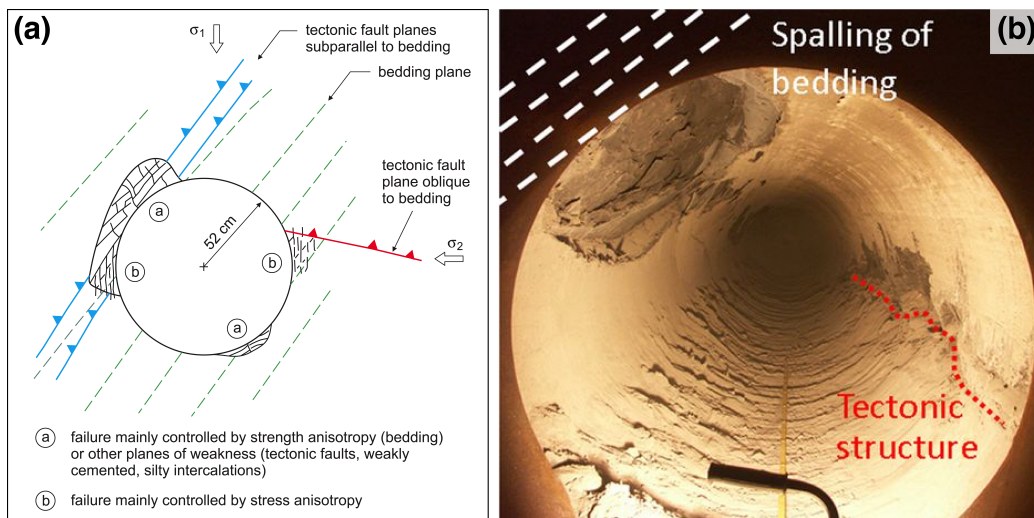


Figure 8: Excavation damage of the HG-A microtunnel: (a) conceptual diagram of the damaged zones and fault traces around the tunnel; and (b) observed damage formation along the tunnel (adapted from Marschall et al. 2006).

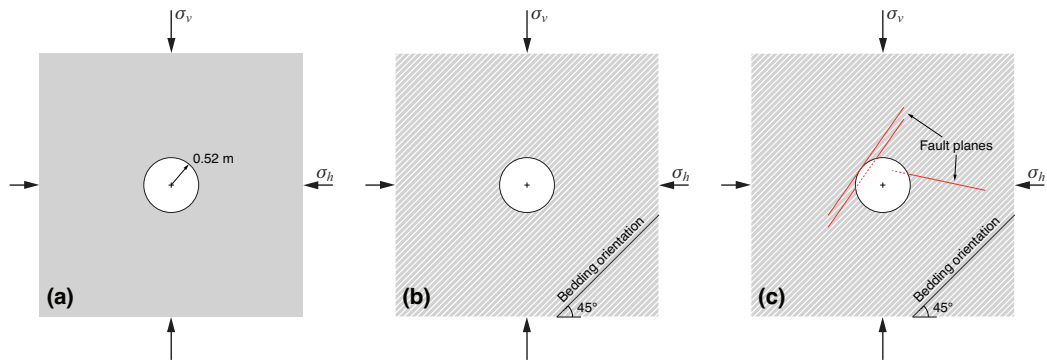


Figure 9: Modeling cases for three different geomechanical settings: (a) Case I for in-plane isotropy and intact formation; (b) Case II for in-plane anisotropy and intact formation; and (c) Case III for in-plane anisotropy and fault formation.

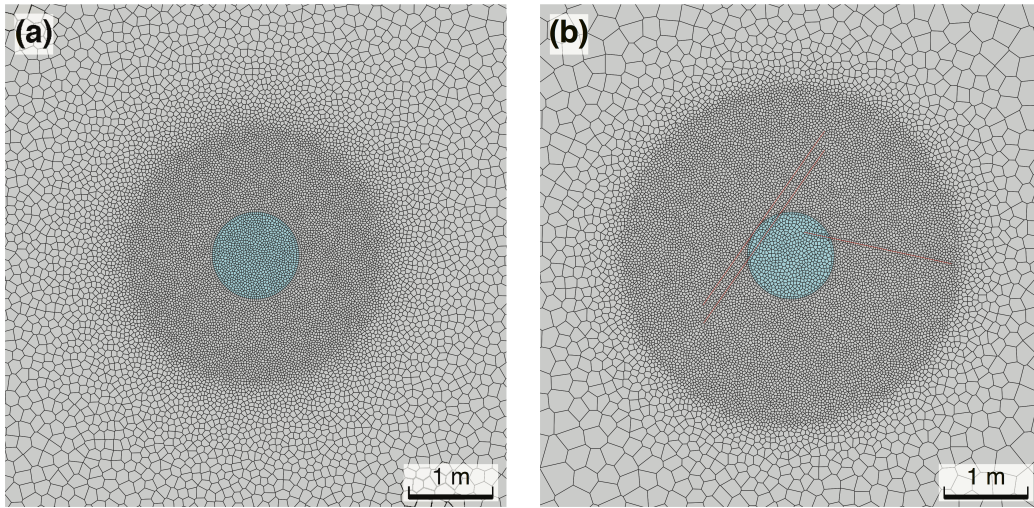


Figure 10: Voronoi discretizations for modeling tunnel geometries in (a) intact formation (Cases I and II) and (b) fault formation (Case III).

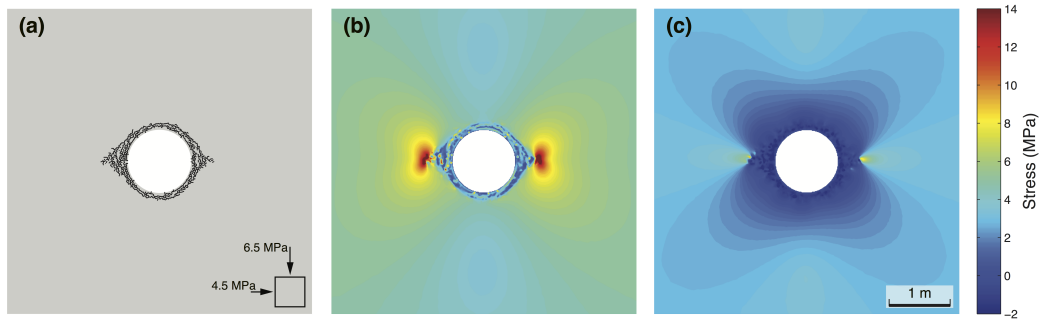


Figure 11: Results at the final stage of the simulation for Case I with in-plane isotropy and intact formation: (a) fracture pattern; (b) major principal stress; and (c) minor principal stress.

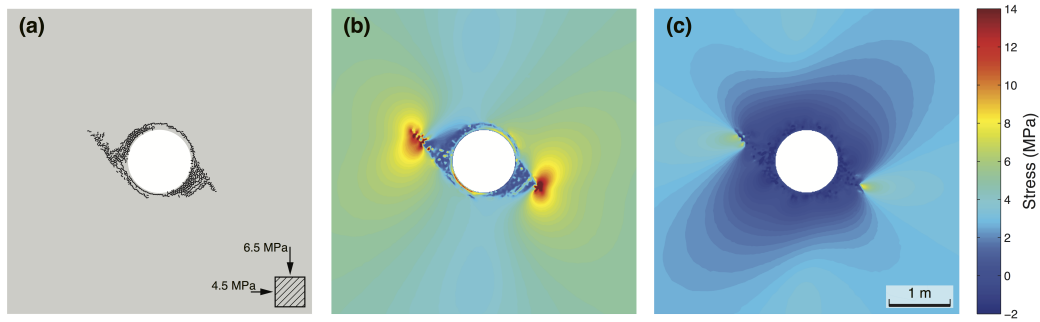


Figure 12: Results at the final stage of the simulation for Case II with in-plane anisotropy and intact formation: (a) fracture pattern; (b) major principal stress; and (c) minor principal stress.

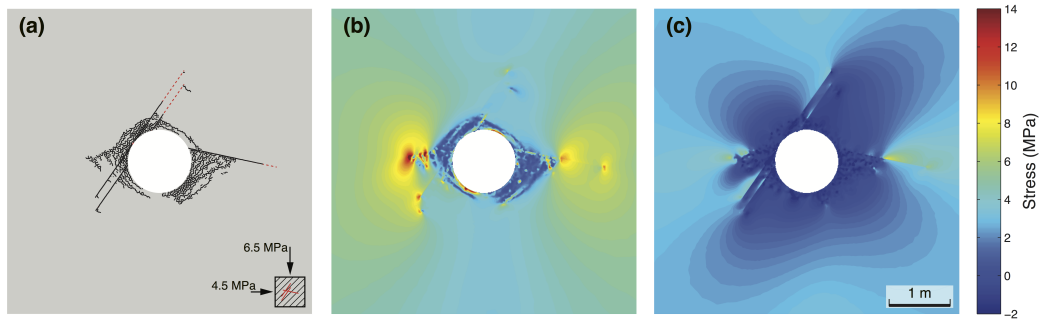


Figure 13: Results at the final stage of the simulation for Case III with in-plane anisotropy and fault formation: (a) fracture pattern; (b) major principal stress; and (c) minor principal stress.